STEEL FIBERS AS SHEAR REINFORCEMENT IN HIGH STRENGTH CONCRETE BEAMS

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ABSTRACT

The purpose was to investigate if steel fibers can replace stirrups as shear reinforcement in high strength concrete beams. Thus, twenty beams were fabricated with various types and amounts of fibers. For comparison reasons, beams reinforced with stirrups and no shear reinforcement whatsoever were also tested. The main variable investigated, in addition to the effect of steel fibers, was the influence of a possible size effect. Hence, beams of three dimensions were produced, 200x250x1500 mm^3, 200x500x3600 mm^3 and 300x700x6000 mm^3. Analysis of the results indicates some favourable aspects concerning the use of steel fibers as shear reinforcement in high strength concrete beams. However, a comparison of test results with some equations, suggested to apply for fiber reinforced concrete beams in shear, shows that there are some uncertainties connected to the design.

keywords: beams; shear; steel fibers; high strength concrete; size effect.

1 INTRODUCTION

In recent times development of new materials and production methods have increased within the field of construction. One example is the use of steel fibers for various applications. Due to its ability to distribute and prevent cracks from appearing steel fibers have proved rather effective as crack controlling reinforcement, particularly in slabs. By replacing parts of the conventional reinforcement by steel fibers a new, more rational way of production has been developed. The favourable properties provided by fibers have lead to increased research efforts aiming at finding other areas of application for steel fiber reinforced concrete. One area where steel fibers may prove effective is as shear reinforcement in concrete beams. Here, the replacement of conventional shear reinforcement may seriously reduce construction time and costs. Quite a few previous investigations /1/, /6/, /13/, /14/, /17/, /18/ and /19/ have been conducted within this field of application, and some design methods applicable on steel fiber reinforced concrete beams in shear have been reported. Results from these studies have indicated some beneficial effects provided by fibers in terms of improving the shear behaviour. However, as most of the shear design procedures have been developed from experimental results, there are reasons to suspect that these will prove to be insufficient, especially regarding the effect of the geometrical dimensions of the beams.

The aim of this study was to investigate the possibility of replacing conventional shear reinforcement, stirrups, with reasonable amounts of steel fibers, 1 vol% or less. As this work was part of a nation wide research program on high strength/performance concrete /10/, /11/, /12/, /15/ and /16/ particular interest was directed towards the shear behaviour of high strength concrete beams. In order to study the accuracy of existing methods of design, results from beams of three different dimensions, 200x250x1500 mm^3, 200x500x3600 mm^3 and 300x700x6000 mm^3, were compared to proposed equations. In this way a measurement of the size effect for steel fiber reinforced beams was provided.

2 TEST PROGRAM

A total of twenty reinforced beams of a relatively high strength concrete were tested. In order to study the effect of an increasing size on the shear capacity, beams of three different dimensions were produced. Moreover, various types and amounts of steel fibers were added to the concrete in order to find an optimal fiber - concrete mixture. Material strength parameters were obtained from uniaxial tensile and compressive tests. As fibers are known to primarily contribute to the post cracking behaviour of the concrete, tensile tests were performed in displacement control. In this way a measurement of the fracture toughness, or residual strength, of the concrete was supplied.

2.1 Specimen Details

The beams were divided into three series with respect to the geometrical dimensions, S, M and L, see Figure 1. The first letter in the notations of Table 1 thus refers to the size of the beam. Plain concrete beams with and without shear reinforcement are denoted STIRRUPS and REF whereas fiber reinforced beams are denoted by the type of steel fibers. Geometrical configurations of the fibers can be found in Table 3.

As indicated in Table 1, one beam in each series contained stirrups as shear reinforcement. The somewhat strange choice of shear reinforcement in these cases, Ø8s130 for series S, Ø8s300 for series M and Ø8s400 for series L, results from the minimum requirement, $s = 0.75d$, according to the Swedish National Code, BBK 94 /5/. Furthermore, the beams contained a substantial amount of longitudinal reinforcement to avoid undesired flexural failures. Also, to prevent an equally undesired anchorage failure from occurring the end zones were provided with relatively high ratios of stirrups, see Figure 1.

Table 1. Specimen details of beams in series S, M and L.

Beam type	Shear reinforcement	Effective depth, d (mm)	a/d	f_{cc}^{cube} (MPa)	f_{ct} (MPa)	Fracture energy, G_f (Nm/m^2)
S-REF	None	180	3.3	113.4	4.15	140
S-STIRRUPS	ϕ8s130	180	3.3	113.4	4.15	140
S-MIX	Mix 1.0 vol%[1)]	180	3.3	118.0	4.98	3310
S-6/015	6/0.15 1.0 vol%	180	3.3	128.4	5.47	1626
S-60/07(I)	60/0.7 0.5 vol%	180	3.3	114.1	4.54	2921
S-60/07(II)	60/0.7 0.75 vol%	180	3.3	114.1	4.53	1985
M-REF	None	410	2.9	98.7	3.77	127
M-STIRRUPS	ϕ8s300	410	2.9	98.7	3.77	127
M-MIX-1	Mix 1.0 vol%[1)]	410	2.9	102.1	4.22	-
M-MIX-2	Mix 1.0 vol%[1)]	410	2.9	102.1	4.22	-
M-6/015-1	6/0.15 1.0 vol%	410	2.9	108.9	4.32	781
M-6/015-2	6/0.15 1.0 vol%	410	2.9	108.9	4.32	781
M-60/07(I)-1	60/0.7 0.5 vol%	410	2.9	98.2	4.23	1370
M-60/07(I)-2	60/0.7 0.5 vol%	410	2.9	98.2	4.23	1370
M-60/07(II)-1	60/0.7 0.75 vol%	410	2.9	107.3	4.49	3117
M-60/07(II)-2	60/0.7 0.75 vol%	410	2.9	85.3	3.97	5256
L-STIRRUPS	ϕ8s400	570	3.0	98.7	3.77	127
L-MIX	Mix 1.0 vol%[1)]	570	3.0	106.6	4.74	2773
L-6/015	6/0.15 1.0 vol%	570	3.0	108.9	4.32	781
L-60/07(II)	60/0.7 0.75 vol%	570	3.0	107.3	4.49	3117

[1)]MIX = mix of 0.5 vol% of 6/0.15 and 0.5 vol% of 30/0.6

2.2 Material Details

The concrete mix was designed in order to reach an average cube compressive strength of about 120 MPa at 28 days of age. Details on the mix design can be found in Table 2. Here, two mix proportions are shown, Mix I and II. The first mix, Mix I, was used in the design of the beams in series M and L whereas the second mix, Mix II, was used in the production of the smaller beams of series S.

Table 2. Mix proportions

Material	Quantity Mix I[1)] kg/m^3	Quantity Mix II[2)] kg/m^3
Cement type Portland std Anl	490	490
Fine aggregate 0 - 8	888	730
Coarse aggregate 12 – 16	787	1152
Silicafume Type Microsilica Densified	48	48
Superplasticizer Type Peramin F	4.6	4.6
Water	157	166
Water / Cement ratio (w/c)	0.32	0.34
Water / Binder ratio (w/b)	0.29	0.31

[1)]Mix I was used for series M and L. [2)]Mix II was used for series S.

Strength variables obtained from compressive and tensile tests are presented in Table 1. Here, the compressive strength was measured on 100 mm cubes whereas the tensile strength was obtained from uniaxial tests on Ø74 mm cylinders, see Figure 2. The fibers were of a drawn wire type. Two of them, 30/0.6 and 60/0.7, were hooked in the ends to provide good anchorage to the surrounding concrete whereas the third one, 6/0.15, was a straight fiber.

Table 3. Material and geometrical data on steel fibers

Types of steel fibers	Geometrical configuration	Aspect ratio (L/D_f)	f_y (MPa)
Dramix® 6/0.15		40	2600
Dramix® 30/0.6		50	1100
Dramix® 60/0.7		86	2600

Longitudinal reinforcement consisted of deformed bars of Swedish Grade Ks600s, with a characteristic yield strength of approximately 590 MPa. For shear reinforcement stirrups of diameters 8 or 10 mm of Swedish grade Ks400s, with a characteristic yield strength of approximately 400 MPa were used.

2.3 Testing Procedure

The test setups and geometrical configurations of the beams are shown in Figure 1. Here, the beams in the first series, S, were subjected to a midpoint load whereas the larger beams of series M and L were subjected to two point loads. The reason for this was to keep the shear span to depth ratios, *a/d*, approximately constant at a value of about 3.0 for all beams. The load was in all cases applied with a deflection velocity of 0,02 mm per second and deflection and load data was registered each third second or each 10 kN.

A number of strain gauges, of an electrical resistance type, were applied to each beam in order to measure the deflections. Two of them were applied on the top of the beam, directly above the supports to measure the support settlements, while the rest of the gauges were uniformly distributed over the beam span. During the loading process cracks were observed and marked out on the beam side and the value of total applied load was inscribed at the end of each crack. Furthermore, the loading was interrupted at each 50 kN in order to allow measurements of crack widths at different stages of the process. In this way, a crack propagation history was recorded on each beam. The duration time for a single test was approximately one hour independently of beam size.

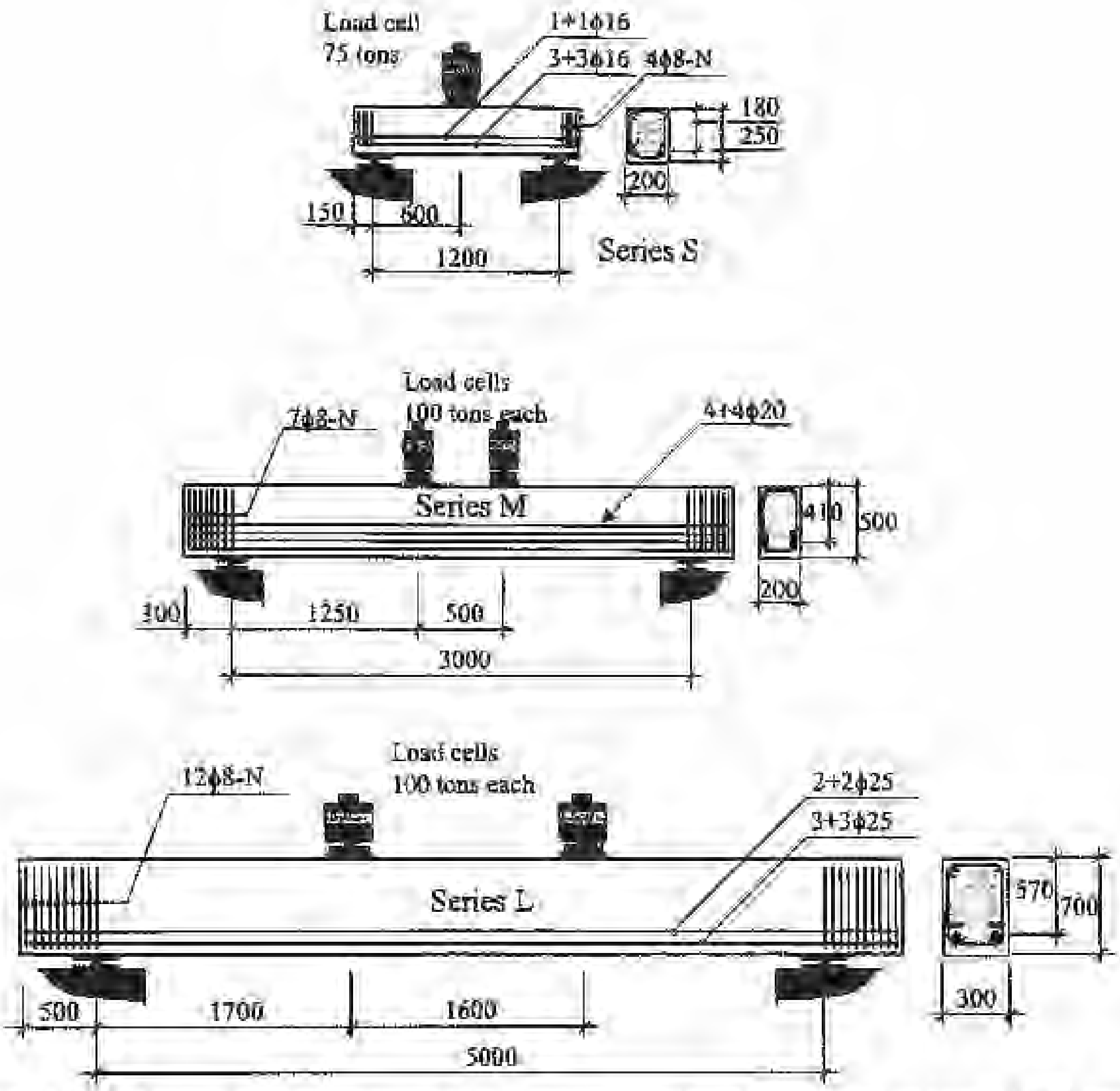


Figure 1. Load setup and geometrical configurations of test specimens in series S, M and L

3 DISCUSSION OF TEST RESULTS

3.1 Tensile Behaviour

The test setup and a specimen for the uniaxial tensile test is shown in Figure 2. As can be seen the specimens were provided with a saddle shaped notch giving a waist diameter in the fracture zone of 55 mm. The reason for this was to be able to control the position of the crack. In this way measurements of the crack width was enabled. The measuring device consisted of four gauges, so called COD:s, positioned around the specimen. By performing the tests in displacement control the post-cracking response of the concrete could be evaluated.

Figure 2. Illustration of the testing device for the uniaxial tensile tests and a specimen for the uniaxial tensile tests.

In Figures 3 (a) and (b) some results from the tensile tests on the different concrete mixes are shown. Here, the tensile stress is shown as a function of the measured crack opening. Thus, the curves represent the post-cracking response, or the residual strength, of the concrete. As can be seen the resistance of the plain concrete rapidly decreased to zero, as soon as the tensile strength was exceeded. The fiber reinforced ones, on the other hand, proved to have substantial reserve capacities, even at relatively large crack openings.

Regarding the effect of different types of fibers on the post cracking response some conclusions can be drawn based on the results. Firstly, it seems as if the 6/0.15 fibers have a certain degree of influence on the pre-peak behaviour, i.e. improves the elastic modulus and tensile strength, see Table 2. A probable reason for this is that these fibers start to act at an early stage of loading preventing micro cracks from extending in the structure. The post-peak performance of this concrete, however, proved to be relatively poor resulting in a quite steep descending curve. In order to get a ductile post-peak behaviour it seems as if longer fibers are required.

The best fracture toughness was provided by the mixed fibers, S-MIX and M,L-MIX. The softening of the concrete reinforced with 60/0,7 fibers did not be quite as good, even though the curves are quite similar in the case of M,L-60/0,7(II) and M,L-MIX. One of the reasons for the somewhat poor results with the 60/0,7 fibers is that the specimens for the uniaxial tensile test may be too small to represent the material correctly. In the case of 60/0,7 fibers the length is greater than the diameter of the fracture zone (55 mm). This may have a certain degree of influence on the results. Also, as the number of fibers in the case of 60/0,7 fibers is less than in the case of shorter fibers the scatter in results is greater.

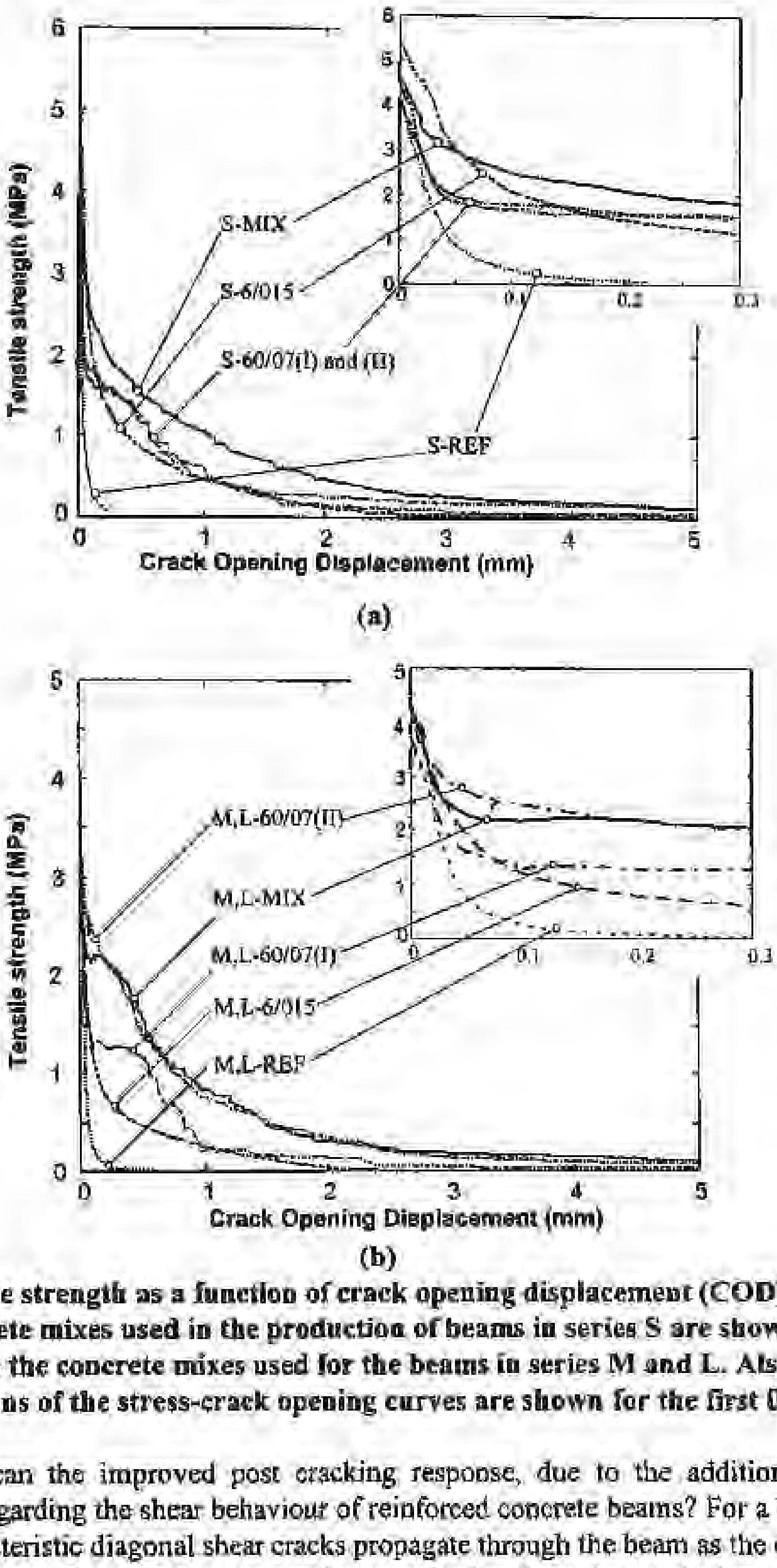


Figure 3. Tensile strength as a function of crack opening displacement (COD). In (a) the concrete mixes used in the production of beams in series S are shown, and (b) shows the concrete mixes used for the beams in series M and L. Also, magnified versions of the stress-crack opening curves are shown for the first 0,3 mm.

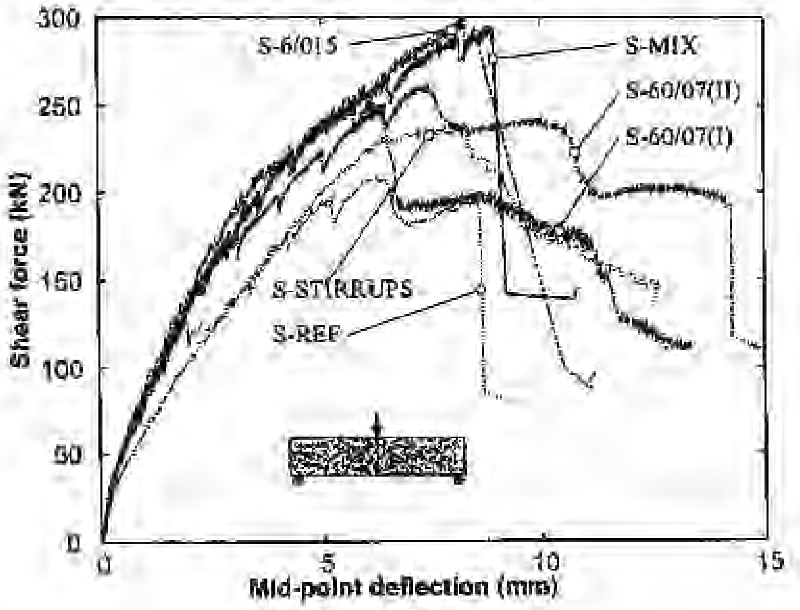
In what ways can the improved post cracking response, due to the addition of fibers, be advantageous regarding the shear behaviour of reinforced concrete beams? For a beam loaded in shear, the characteristic diagonal shear cracks propagate through the beam as the concrete tensile strength is exceeded. For a plain concrete the contribution from the cracked parts of a structure to the shear resistance is often neglected. In the case of a fiber reinforced concrete, however, a considerable contribution may be expected from the cracked parts. This may be attributed to the ability of the fibers to bridge and carry load from one side of a crack to the other. In this way, fibers may contribute to the shear resistance of the beam even at relatively large crack openings and thus contribute to an increased failure ductility.

3.2 Shear Capacities

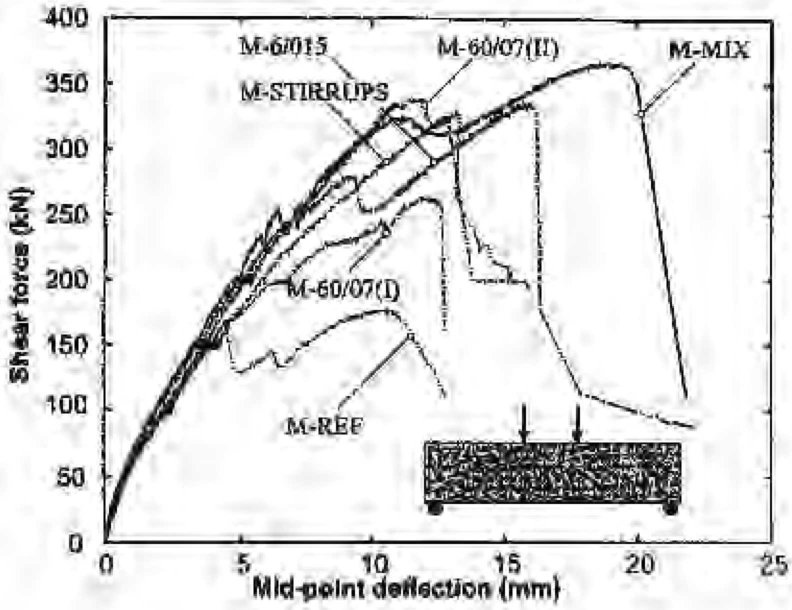
In Figures 4 (a), (b) and (c) the shear force is plotted as a function of the mid point deflection. As the beams in series S were subjected to a mid-point load the shear force was obtained from the following relation: $V = P/2$, where P represents the applied load. The larger beams of series M and L, however, were subjected to two uncoupled point loads, and hence, the shear forces were calculated as follows:

$$V = P_{max}(a+c)/L + P_{min}a/L \tag{1}$$

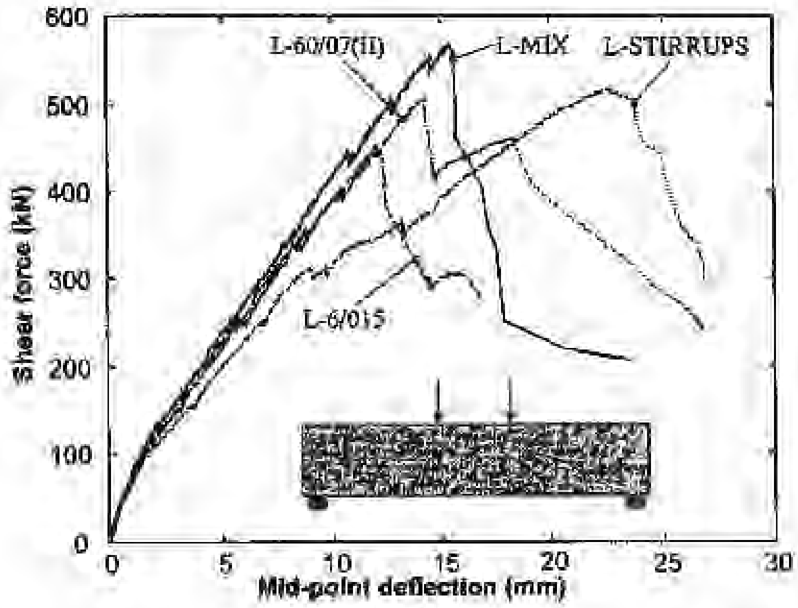
which represents the actual shear force in the beams. In the above relation P_{max} was taken as the larger of the two loads and P_{min} the smaller. L is the beam span, a the shear span and c the distance between the loads.



(a)



(b)



(c)

Figure 4. Shear force as a function of mid point deflection for (a) beams in series S, (b) beams in series M and (c) beams in series L.

From the results it is quite clear that the ultimate shear capacities were positively affected by the steel fibers. As a comparison the addition of 1 % by volume of 6/0.15 steel fibers to beams in series S, S-6/015, increased the ultimate strength by 42 % whereas the corresponding value for stirrups, S-STIRRUPS, was only 12%. The effectiveness of steel fibers for beams in this series may be illustrated by the fact that the beams S-MIX and S-6/015 exhibited crushing of the compression zones at peak load, which limited the post-peak performances of the two beams and they failed in quite abrupt fashions. This indicated the ability of the fibers to resist the increasing levels of tensile stresses across the diagonal cracks until the concrete compressive strength limited further increases of the load.

The other fibrous beams in this group, S-60/07(I) and (II), did not reach ultimate loads of the same magnitude. On the other hand, these beams exhibited far more ductile post-peak behaviour. Thus, it is reasonable to assume that the content of fibers in these cases, 0.5 and 0.75 % by volume, was too low to provide the same resistance against shear stresses. As a ductile post peak behaviour is desirable, however, the fiber amounts in the earlier mentioned beams, S-MIX and S-6/015, were possibly too high.

The fiber reinforced beams of series M, see Figure 4 (b), were able to resist in average 78 % higher ultimate stresses than the reference beam, M-REF, with a maximum value for M-MIX of 107 %. The beam reinforced with stirrups, M-STIRRUPS, had a corresponding value of 86 %. When comparing the ultimate capacities of beams in series S and M it seems as if the need for shear reinforcement increases with increasing beam size. This conclusion is based on the fact that the difference between the shear capacity of the reference beam and the fiber reinforced beams is considerably greater for the larger beams.

The fiber reinforced beams in series L, see Figure 4 (c), showed a considerably stiffer behaviour up to maximum load than the conventionally reinforced beam, L-STIRRUPS. It is, however, important to mention that the amount of stirrups was relatively low, which makes a comparison a bit unfair. The beam reinforced with the shortest fibers, L-6/0.15, failed at a considerably lower load than the other fibrous beams in this series. Thus, these fibers were evidently too short to seriously affect the load carrying capacity of the beam. A conclusion that may be drawn here is that longer fibers, which are better anchored to the concrete, are needed when the beam size increases. This to be able to carry the relatively higher load that is released when a larger beam cracks. For the conventionally reinforced beam the sudden failure was caused by the fracture of the stirrups that crossed the critical diagonal crack.

3.3 Effect of Fiber Type

An important factor to take into account when evaluating the shear performance of fiber reinforced beams is the type of fibers. Generally a mix of 6/0.15 and 30/0.6 steel fibers proved to be the most effective reinforcement. For the small beams of group S the short straight fibers, 6/0.15, were as effective as the mixed fibers. For larger beam sizes, on the other hand, these fibers were presumably too short to give the same contribution to the shear resistance, resulting in a complete pull-out of all the fibers at an early stage of loading.

The relatively poor performance of the beams reinforced with 60/0.7 fibers may be due to one of the following aspects, or a combination of them. Firstly, the amount may have been too low to seriously enhance the ultimate capacities and provide ductile post-peak behaviour. This would explain the fact that beams S-60/07(I) and (II) performed fairly well whereas the corresponding beams in series M and L, M-60/07(I) and (II) and L-60/07(II), exhibited quite abrupt failures at relatively early stages of loading.

The second aspect that might have affected the performances of these beams is the difficulty in getting a uniform distribution of the fibers in the matrix. As the same amount of long fibers, 60/0.7, corresponds to a lower number of fibers than in the case of 6/0.15 or 30/0.6 fibers, the individual distance between fibers localised in the crack zone would be much less for the longer fibers. Thus, if the shorter fibers prove to be sufficiently anchored to the structure the incentive for using a longer type of fibers would vanish. This may very well be the case for high strength concrete where the bond quality can be expected to be better than for a lower strength concrete.

However, a comparison of the shear capacities, see Figures 4 (a), (b) and (c), implies that the effectiveness of the 60/0.7 fibers increases with increasing beam size. In other words, the requirements on the fibers become greater as the beam dimensions increase. For small beams a shorter type of fibers may be sufficient, whereas for larger beams, fibers that are better anchored and have a higher strength are required.

3.4 Size Effect

The results from the shear capacities of the beams, presented previously, indicate the necessity of bringing about a discussion on the structural size effect and its importance for fiber reinforced structures. In particular, it could be noticed that the failures of the smaller beams were considerably more ductile as compared to the larger beams. An indication of the difference in brittleness is given by the crack patterns on the side faces of the fractured beams. Some photos of the beams with mixed fibers, S-MIX, M-MIX-1 and L-MIX, are shown in Figure 5. The large number of visible diagonal cracks on the side face of the smallest beam, S-MIX, gives an indication of the failure ductility of this beam. On the other hand, for the largest beam, L-MIX, only one large diagonal crack is visible. This means that the residual strength of the fiber concrete was not sufficient to resist the large impulse load that was released as the crack started to develop. As a consequence the beam failed in a considerably more brittle fashion.

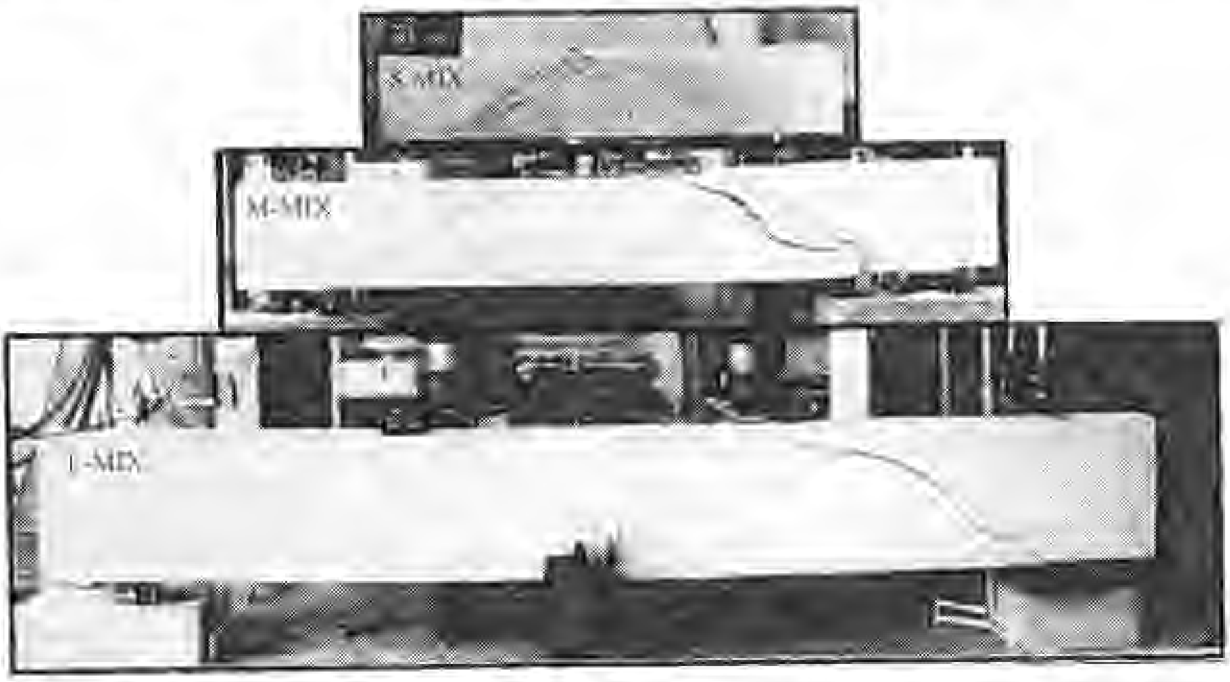


Figure 5. A comparison between crack patterns of beams containing the mixed type of fibers, S-MIX, M-MIX-1 and L-MIX. Noticeable is that for the smallest beam there are an amount of diagonal cracks, whereas for the largest beam there is only one critical crack.

By studying a fracture mechanics approach it may be possible to explain how an increasing beam size influences the ability of the fibers to contribute to the shear resistance. To define some basic fracture mechanical features results from the uniaxial tensile test on S-MIX is shown in Figure 6. Up to the point where the tensile strength of the concrete is exceeded, the structure is elastically strained, i.e. the deformations are evenly distributed over the structure. When the tensile strength f_{ct}, is exceeded a fracture zone is formed. At this stage all deformations are concentrated to this fracture zone while other parts of the structure are elastically unloaded.

A brittleness number /2/, /15/ and /16/ is commonly defined as the ratio between stored elastic energy and dissipated fracture energy. From this definition follows that the brittleness increases when the structural size, L, increases, if the material characteristics are kept constant.

$$Brittleness = \frac{Elastic\ energy}{Fracture\ energy} = \frac{f_{ct}^2 \cdot L}{2E \cdot G_f} \tag{2}$$

In other words, the size effect is due to the release of strain energy from the structure into the fracture zone as the cracking zone extends. The larger the structure the greater is the energy release and the more fracture energy is needed to avoid a sudden failure.

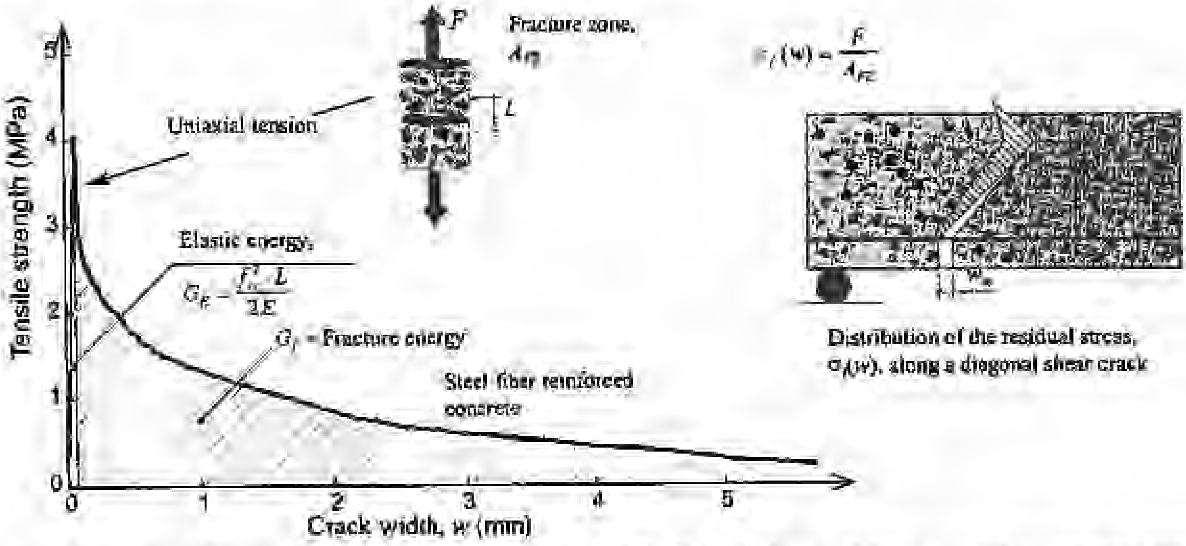


Figure 6. Comparison of stress - strain relationship for plain and fiber reinforced concrete in tension. Also, some fracture mechanical properties are shown. G_E equals the stored energy per unit area and G_f is the fracture energy per unit area.

In what ways do the above defined brittleness characteristics apply to the beams tested in this investigation? Earlier studies /3/, /4/ have shown that the brittleness of a beam loaded in shear to some degree is proportional to the effective depth, d. From the brittleness definition discussed above this would imply that the larger the beam the greater amounts of strain energy is stored in the structure. When, eventually, the tensile strength is exceeded and a critical diagonal crack forms, see Figure 6, the residual tensile stresses acting across the crack have to resist the impulse load that is connected to the energy release. Hence, the requirements on the tensile properties of the fiber concrete will increase as the beam size increases.

Based on the fracture energy, G_f in Table 1, some conclusions regarding the effectiveness of different types of steel fibers can be drawn. As can be seen in Figure 4 there was practically no difference in ultimate shear capacity between S-MIX and S-6/0.15 whereas the shear capacity differed considerably between L-MIX and L-6/0.15, 570 kN compared to 450 kN. For the small beams the fracture energy obtained from uniaxial tensile tests is about twice as great for S-MIX than for S-6/0.15, 3300 compared to 800 Nm/m^2. Obviously this did not have much of an influence in this case. This means that the amounts of fibers were presumably unnecessarily high for these beams as the structures had a certain amount of ductility in themselves.

If the fracture energy of the larger beams is looked upon it can be seen that L-MIX had about 3,5 times greater fracture energy than L-6/0.15. Consequently, the first mentioned beam reached a shear capacity that was about 30 % greater than for L-6/0,15. Nevertheless, none of the beams in the largest series experienced a particularly ductile failure. This would imply that larger amounts of fibers are needed to be able to withstand the large impulse load connected to the energy release into the fracture zone.

4 COMPARISON OF TEST RESULTS WITH SHEAR DESIGN EQUATIONS

The previously presented results have most definitely proved that it is possible, at least to some degree, to replace stirrups by steel fibers in reinforced concrete beams. However, for fibers to be a competitive alternative to stirrups it is necessary to find a good and simple design method. Thus, in this section some theoretical models, see Table 4, for shear prediction of fiber reinforced concrete beams are compared to the results of the tested beams.

Table 4. Equations for estimating the shear capacity of steel fiber reinforced beams

Designations	Equations
Ashour /1/	$V_u = \left[\left(0.7 \cdot \sqrt{f_{cc}} + 7F_f\right)\frac{d}{a} + 17.2 \cdot \rho_s \frac{d}{a}\right] \cdot wd$
Imam /13/	$V_u = 0.6 \cdot \frac{1+\sqrt{5.08/d_a}}{\sqrt{1+d/25d_a}} \cdot \sqrt[3]{\rho_s \cdot \left(1+4F_f\right)} \cdot \left[f_{cc}^{0.44} + 275 \cdot \sqrt{\frac{\rho_s \cdot \left(1+4F_f\right)}{(a/d)^5}}\right] \cdot wd$
Narayanan /14/	$V_u = \left(0.24 \cdot \left(\frac{f_{ct}}{20-\sqrt{F_f}} + 0.7 + \sqrt{F_f}\right) + 80\rho_s \frac{d}{a} + 1.7F_f\right) \cdot wd$
S.C.A /18/	$V_u = \left[\xi\left(1+50\rho_s\right) \cdot 0.30 f_{cr} + 1.7 \cdot F_f\right] \cdot wd$
Casanova /6/	$V_u = 0{,}2 \cdot \sqrt{f_{cc}} \cdot \sqrt[3]{\rho_s} \cdot W \cdot H + 0{,}9 \cdot b \cdot d \cdot \sigma_p(w_m)$

In the Casanova /6/ shear model the contribution of the steel fiber concrete to the shear capacity is based on the average post-cracking residual strength, $\sigma_p(w)$, of the fiber concrete. This model assumes that a critical diagonal crack has formed and led to a block mechanism, see Figure 7. The crack is assumed to be inclined 45 degrees and the width varies linearly from a maximum at the level of the longitudinal reinforcement to zero at the bottom of the compression zone. By integrating the post-cracking tensile strength, $\sigma_f(w)$, along the crack and projecting it vertically, the average residual strength between 0 and w_m is obtained as:

$$\sigma_p(w_m) = \frac{1}{w_m} \cdot \int_0^{w_m} \sigma_f(w)dw \tag{3}$$

where

w_m = the maximum crack width

$\sigma_p(w_m)$ = mean value of the post cracking residual strength between a crack width of zero and w_m.

It is further suggested that the estimation of the maximum crack width, w_m, is based on the strain in the longitudinal steel and the effective depth of the beam, such that:

$$w_m = \varepsilon_s \cdot 0{,}9d \tag{4}$$

The limiting value of the steel strain is proposed to 1 %, which corresponds to the maximum allowed strain according to the French design rules. For the beams in this study this would give ultimate crack openings of 1,62 mm for beams in series S, 3,69 mm for beams in series M and 5,13 mm for beams in series L. Thus, the fact that w_m depends on the effective depth, d, means that the maximum crack widths increase with increasing beam sizes. As a consequence the average residual strength of the fiber concrete, $\sigma_p(w_m)$, will decrease with increasing size.

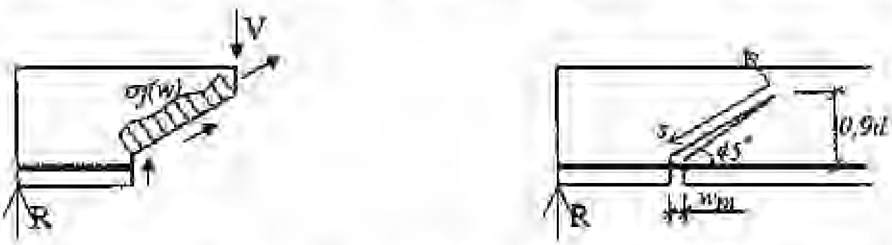


Figure 7. Basic principles for estimating the contribution of fiber concrete to the shear capacity according to Casanova /6/.

From Figure 8, where the ratio calculated over experimental shear force, V_{cal}/V_{exp}, has been plotted to the effective shear area, $b \cdot d$, it is quite clear that the predictive formulas did not describe the shear capacities with great accuracy. In general, it seems as if the formulas tend to underestimate the shear capacities of the smaller beams of series S, make fairly good predictions for beams in series M and slightly overestimate the capacities of the largest beams of series L. This development is quite dangerous considering that beams with depths of up to 1000 mm and above are frequently used in design.

The best predictions were obtained by the Imam formulation. This shear equation has been developed for steel fiber reinforced high strength concrete beams using a fracture mechanics approach. A difficulty in using the Casanova shear model, apart from performing uniaxial tensile tests, is to decide the maximum crack width, w_m. As the post-cracking residual strength depends upon this value the contribution of steel fiber concrete to the shear capacity, V_f, may not be correctly estimated. On the other hand, as this model consists of two parts, a structural part and a fiber part, it is not necessarily the contribution from the steel fibers that is misjudged.

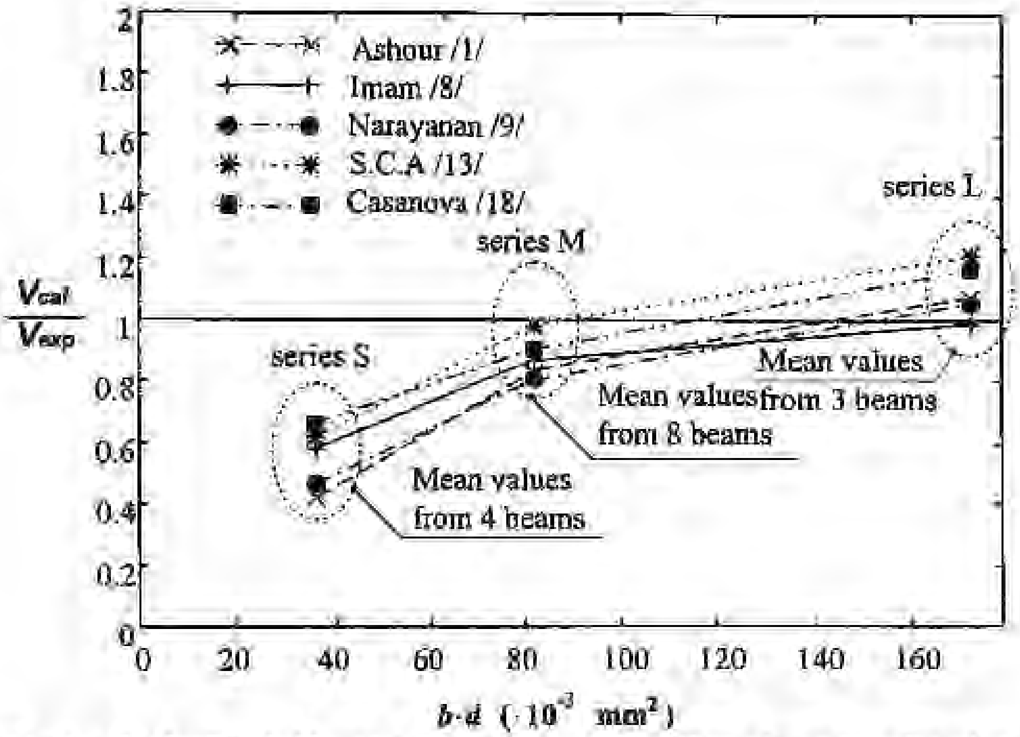


Figure 8. Ratio of calculated, V_{cal}, over experimental, V_{exp}, shear force as a function of the effective shear area, $b \cdot d$, for some design formulations.

One of the main reasons for the difficulties in finding good predictive formulations to describe the shear capacities may be that the empirical relations used to estimate the shear contribution of the concrete part, V_c, have been developed for normal strength concrete. As there are differences between normal and high strength concrete, in particular regarding the brittleness of the materials, the relations may not be valid. Also, the size effect seems to play a decisive role for the shear capacity. Thus, in order to find reliable methods to predict the shear capacitiy of fiber reinforced beams it is important to take the effect of an increasing size under consideration.

5 CONCLUSIONS

The purpose of the present study is to explore different aspects of the shear performance of steel fiber reinforced high strength concrete beams. In particular, the effect of various types and combinations of steel fibers, and moreover, the influence of geometrical dimensions of the beams, have been in focus here. To accomplish these objectives, an experimental program, consisting of twenty beams of various dimensions reinforced with different types of shear reinforcement, has been performed. From the results of this study the following conclusions can be drawn :

1) By adding steel fibers in relatively small concentrations, 1 vol% or less, it is possible to reach shear capacities of the same order as in the case of conventional shear reinforcement, at least for small beams with an effective depth of about 200 mm.

2) In general, a mix of short, straight fibers, 6/0.15, and longer fibers, 30/0.6, that were hooked in the ends, provided the best contribution to the shear resistance of the beams. A possible explanation is that the small fibers start to act at an early stage, arresting and distributing micro cracks. When a critical shear crack forms, however, the small fibers can not contribute at the same degree anymore. Instead, the longer fibers that bridges the crack take over, and thus provide an increased failure ductility.
3) Concerning the effect of the dimensions of the beams, it seems as the larger the beam the larger amounts of steel fibers are needed to be able to carry the excessive load that is released as a diagonal shear crack forms. One explanation for this is that a larger structure has a relatively higher brittleness than a smaller. This means that the requirements on the fibers increase with increasing beam size.
4) Some shear design equations, suggested to apply for steel fiber reinforced concrete beams, were compared to the experimental results from this study. In general, this comparison showed that there are uncertainties regarding the design of such beams. The fact that the formulas tend to overestimate the capacities of the largest beams, series L, is particularly dangerous. The best predictions were obtained with the equation suggested by Imam /13/, which also considered the effect of an increasing size fairly well.

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NOTATIONS

a	=	shear span (mm)
d	=	effective beam depth (mm)
f_{cc}	=	concrete compressive strength (MPa)
f_{ct}	=	concrete tensile strength (MPa)
F_f	=	fiber factor = $\rho_f d_f L_f / D_f$
d_f	=	bond factor = 0.5 for round, 0.75 for crimped and 1.0 for indented fibers
D_f	=	diameter of fiber (mm)
L_f	=	length of fiber (mm)
ρ_f	=	amount of fibers (vol%)
H	=	beam depth (mm)
V_u	=	ultimate shear capacity (kN)
w	=	beam width (mm)
ρ_s	=	reinforcement ratio

$$\xi = 1.4 \quad \text{for } d \leq 200 \text{ mm}$$
$$= 1.6 - d \quad \text{for } 200 < d \leq 500 \text{ mm}$$
$$= 1.3 - 0.4d \quad \text{for } 500 < d \leq 1000 \text{ mm}$$

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